

Vulnerability Assessment and Lifecycle Analysis of an Existing Masonry Arch Bridge

Raffaele Cucuzza^a, Angelo Aloisio^b, Roberta Di Bari^c, Marco Domaneschi^{a,*}

^a*Politecnico di Torino, Department of Structural, Geotechnical and Building Engineering (DISEG), Corso Duca degli Abruzzi, Turin, Italy*

^b*Department of Civil, Construction-Architectural and Environmental Engineering, Università degli Studi dell'Aquila, L'Aquila, 67100, Italy*

^c*University of Stuttgart, Institute for Acoustics and Building Physics, Department Lifecycle Engineering (GaBi), 70569, Stuttgart, Germany*

Abstract

This paper presents a comprehensive study on structural verification and proposed improvements for an ancient masonry arch bridge. The research encompasses multiple stages, starting with a detailed survey utilizing LiDAR and aerial techniques to collect precise data on the bridge's condition. Mechanical inspections assess deterioration levels and identify critical areas requiring attention. Based on the gathered data, a structural model of the bridge is developed, considering geometric parameters and material properties. The model undergoes rigorous verification procedures to assess its seismic response and ensure compliance with safety standards. The analysis includes identifying potential collapse mechanisms, such as the formation of plastic hinges, which can lead to structural failure. The study proposes structural improvements to enhance the bridge's performance and safety. These interventions specifically target vulnerabilities identified during the verification process. They include additional support elements, reinforcing critical areas, and utilising advanced materials to improve tensile strength and durability. The feasibility of the proposed retrofitting solutions will be proved by performing a cost analysis, while environmental impacts are evaluated through an environmental assessment (Lifecycle Assessment - LCA) in which all the main bridge refurbishment phases have been included. Finally, some of the advantages of bridge refurbishment measures are shown by comparing the proposed intervention with a bridge reconstruction in terms of costs and environmental impacts. The outcomes of this study contribute to the existing knowledge on the rehabilitation of masonry arch bridges and serve as a valuable reference for engineers and practitioners involved in preserving architectural heritage.

Keywords: Masonry bridge; LiDAR; Structural analysis; Rehabilitation; Architectural heritage; Cost analysis; Lifecycle Assessment

1. Introduction

For centuries, masonry bridges have been integral to transportation infrastructures, witnessing our engineering heritage [1–3]. With their unique characteristics and historical significance, these structures require careful attention to ensure their preservation and structural integrity [4–6]. In recent years, advancements in surveying techniques, modelling technologies, and structural analysis have enabled engineers to conduct detailed assessments and implement targeted interventions to rehabilitate existing masonry bridges [7–10]. Recent advancements also include structural health monitoring practices [11–14] to assess the condition and behaviour of masonry arch bridges in near real-time [15–18]. These methods involve sensors, non-destructive testing methods, and data analysis techniques to detect and evaluate structural deterioration, allowing timely intervention and maintenance [19–24]. The structural assessment, also based on advanced monitoring, is the proxy of rehabilitation and retrofitting [25]. Rehabilitation includes using advanced materials [26], reinforcement techniques, and innovative retrofitting methods. The rehabilitation and retrofitting are mostly related to their seismic vulnerability [27, 28]. Masonry arch bridges are vulnerable to seismic events, and understanding their seismic behaviour is crucial for ensuring their safety [29, 30]. In the last years, several researchers have conducted seismic analysis, evaluated collapse mechanisms [5, 31], and developed retrofitting techniques to improve the bridges’ resistance to earthquakes, minimizing potential damage and ensuring the safety of the structures and their users [32–34].

Nonetheless, the rehabilitation often conflicts with sustainability and conservation issues [35, 36]. Efforts are made to incorporate sustainable practices and conservation principles into the preservation and maintenance of these bridges [37]. This involves developing environmentally friendly materials, strategies for energy efficiency and approaches that minimize the impact on the surrounding environment.

In recent years, there has been a growing body of research proposing advanced approaches that integrate high-fidelity modelling [38–40] and reliability analysis techniques for masonry arch bridges [41–45]. However, these advanced analyses often remain confined to scientific discussions, while practitioners rely on more straightforward approaches based on standard regulations [46].

In light of this, there is significant value in contributing case studies to the scientific literature. Although case studies may not possess groundbreaking novelty, they serve as a crucial foundation for generalization and applying more advanced methods. By examining real-world examples, case studies provide practical

*Corresponding author. angelo.aloisio1@univaq.it

Email addresses: raffaele.cucuzza.polito.it (Raffaele Cucuzza), angelo.aloisio1@univaq.it (Angelo Aloisio), roberta.di-bari@iabp.uni-stuttgart.de (Roberta Di Bari), marco.domaneschi@polito.it (Marco Domaneschi)

insights into the challenges and complexities associated with the conservation and rehabilitation of masonry
30 bridges.

While advanced modelling and analysis techniques continue to evolve [47–50], case studies are vital in
bridging the gap between theory and practice. They shed light on the practical issues that practitioners
encounter and help refine the implementation of innovative approaches. Researchers can identify the unique
circumstances and factors influencing decision-making in preserving and rehabilitating masonry bridges by
35 studying specific cases [51].

This paper focuses on the modelling and structural verification of an existing masonry bridge, aiming to
comprehensively assess its condition, evaluate its seismic performance, and propose potential improvements
to enhance its safety and durability. The research encompasses a multi-stage process that begins with a de-
tailed bridge survey, incorporating advanced techniques such as LiDAR and aerial surveys to capture precise
40 geometric data. The collected survey data are used to develop an accurate and representative structural
model of the bridge. Based on the findings of the structural verification, a set of proposed improvements is
presented to enhance the bridge’s performance. These proposals encompass a range of measures, including
introducing additional support elements, reinforcing vulnerable sections, and using advanced materials to
improve strength and durability. The analysis of the proposed strategies is carried out in a holistic per-
45 spective, by evaluating also costs and environmental impact assessments. This enables feedbacks on the
economic and environmental potential of such measures, especially when compared with the demolition
and the reconstruction of a new bridge. The outcomes of this study are expected to provide insights and
recommendations for engineers, preservation experts, and decision-makers involved in the maintenance and
preservation of masonry bridges.

50 **2. The masonry arch bridge**

The bridge under investigation, shown in Figure 1, is a masonry arch bridge located in Southern Italy.
It consists of 193 meters long overall and about 9 meters wide. The bridge comprises eight segmental arches
supported by piers and abutments at the terminal zones. Each arch has a uniform thickness of about 80
centimeters. The minimum span between arches is 11.20 meters for the end arches, while the central arches
55 have a maximum span of approximately 12.50 meters. The arches are constructed using heterogeneous stone
materials connected with low-quality mortar.

Furthermore, the bridge has a maximum height of approximately 12 meters. Over time, the piers have
been reinforced with locally sourced stonework buttresses. Nonetheless, the connection between the piers

and buttresses was not carefully designed to ensure proper adherence between the two elements.

60 The modifications made to the bridge at different times have resulted in a heterogeneous material composition, which poses challenges for structural evaluation. Additionally, the modifications were not carried out following established masonry techniques, particularly to the alignment of mortar joints.



Figure 1: (a) Perspective view of the structure; (b) Bridge before the reinforced concrete interventions.

Table 1: General characteristics of the bridge

Characteristic	Value
Overall Length	193 m
Width	9 m
Number of Arches	8
Arch Thickness	80 cm
Minimum Span (End Arches)	11.20 m
Maximum Span (Central Arches)	12.50 m
Material Composition	Heterogeneous Stone
Mortar Quality	Low-quality
Maximum Height	12 m

At the end of the twentieth century (likely in the 1980s), works were carried out to replace the roadway. Reinforced concrete beams in the transversal directions were introduced with a variable but average spacing
65 of approximately 3.20 m. The cross-section of these beams is square, measuring about 30 cm x 30 cm, but it varies when reaching the bridge overhangs as a 30 cm-thick slab is poured over them. Additionally, safety barriers, consisting of concrete walls and steel handrails, were added to prevent pedestrians from falling. The insertion of the reinforced concrete beams caused partial destruction of the tympanums, resulting in the degradation of the structure.

70 3. Terrestrial and Aerial Survey

The authors used LiDAR and drones for the geometric survey.

3.1. LiDAR

LiDAR (Light Detection and Ranging) is a well-established remote sensing technology to assist in the mapping, monitoring, and evaluating of surfaces and objects. The LiDAR technology enables scientists and professionals to precisely and accurately examine natural and man-made environments flexibly.

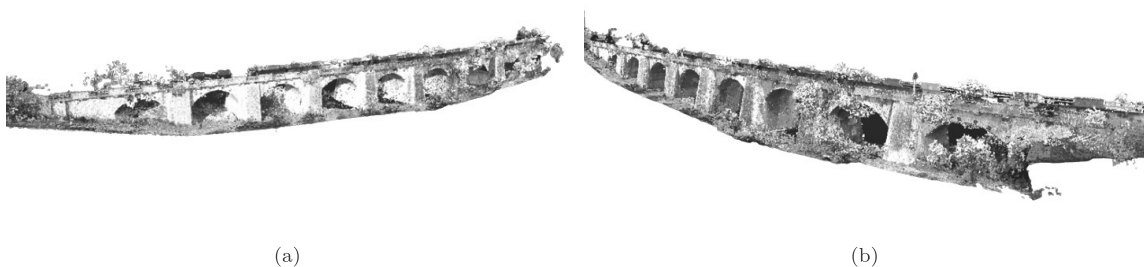


Figure 2: Graphic representation of the bridge from the LiDAR survey.

For the survey of the subject under examination, a 3D laser scanner with a compensating level within 5° and activated integrated barometric and GPS sensors were utilized. Approximately 60 scans were performed, and several known points were selected to align the terrestrial and aerial surveys. The intrusive presence of vegetation, especially in areas adjacent to the roadway platform, such as the tympanums, caused inaccuracies in the instrument collimation. This was addressed by employing convergent setups, which minimized shadow areas in the point cloud generation process.

The point cloud was processed and optimized to represent the bridge graphically, see Figure 2.

3.2. Aerial survey

In recent years, aerial survey techniques using drones are widespread. Drones are equipped with a camera and a wireless control system, allowing long-distance aircraft operation through a joystick from the ground. To conduct the survey, four flights were conducted on separate days, see Figure 3. High-resolution videos were recorded during these flights, capturing the structural details in 4K resolution. This approach proved particularly valuable in identifying areas of structural degradation where terrestrial access was limited or not feasible.

In addition to the videos, the drone captured approximately one hundred photos for photogrammetric point cloud processing, partially shown in Figure 4. An aerial photogrammetric reconstruction complements



Figure 3: Plan of one of the four flights of the drone.

the point cloud derived from the terrestrial LiDAR survey to ensure a more comprehensive survey. This integration allows the point cloud to cover the entire structure, enhancing the completeness of the survey.

3.3. Geometric survey

95 As previously mentioned, the surveying techniques employed were necessary to produce the essential graphical outputs for the graphic reconstruction of the structure, using the AutoCAD software [52]. The graphical outputs are the plan, the elevation and the cross-section views.

As observed in Figure 5(a), the structure does not exhibit significant heterogeneity in its structural components. The arches may appear uniformly lowered due to pier widening, but this is not the case.
 100 In Figure 5(b)-(c), irregularities in the shape of the arches can be seen. The arches display two different types and exhibit distinct structural behaviours due to interventions carried out in the twentieth century. Furthermore, the plan view of the structure is asymmetric, as not all the pier widening interventions have been equal on both sides.

Combining the plan, the elevation and the cross-section views led to a three-dimensional graphical
 105 model. It is worth discussing some aspects encountered in reality that were simplified in this model, as it was exported for integration into the Midas CIVIL © software [53] structural analysis.

As observed from the surveys, the bridge has a slight longitudinal slope on both sides, converging towards the centre of the structure. However, regarding the structural model, it is not meaningful to consider such a slight longitudinal slope as it has negligible influence on stress calculations. Therefore, this aspect was
 110 neglected in the three-dimensional model. However, due to the various differences in terms of geometric details (e.g. different piers' height, different spans' length and shallowness grade of the vaults) and/or

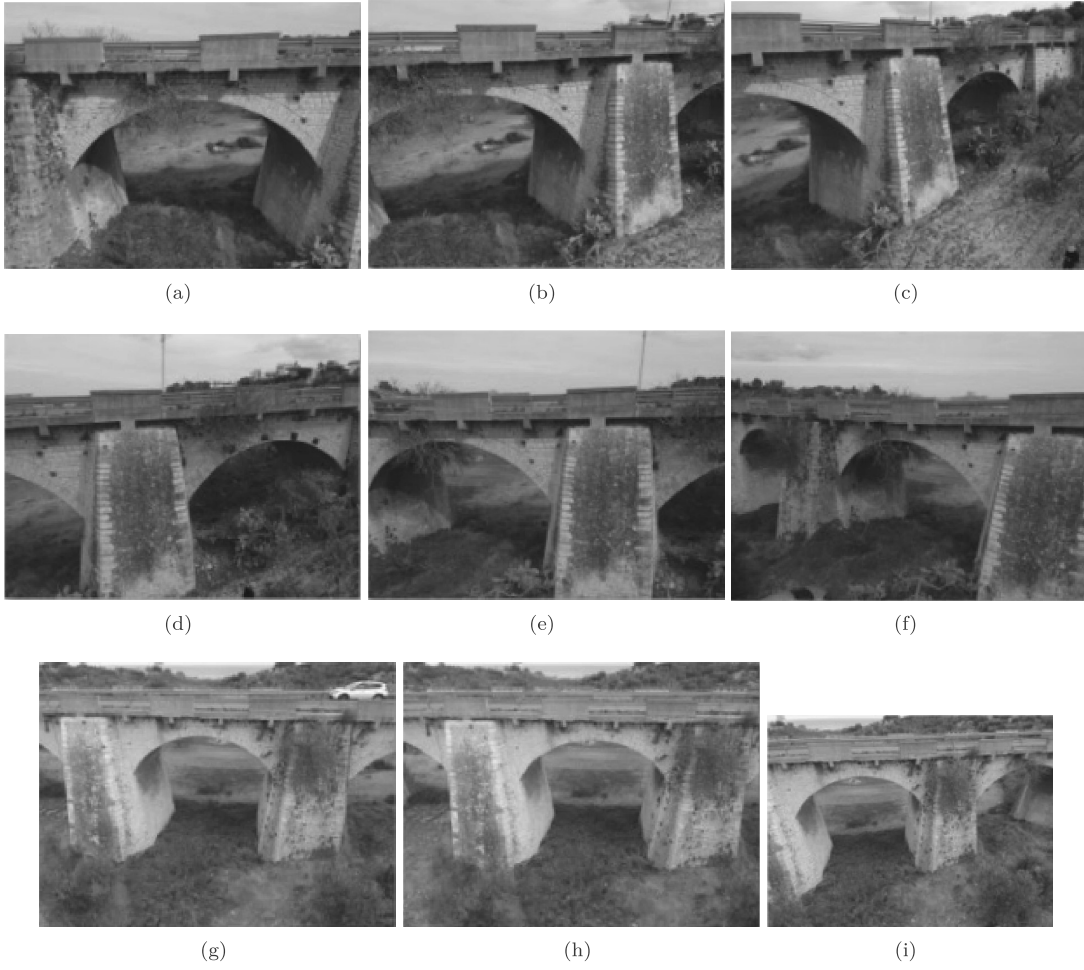


Figure 4: Photos taken during the aerial survey.

mechanical properties (e.g. different masonry properties and different levels and extensions of pathologies), a detailed modelling of each span of the bridge has been considered.

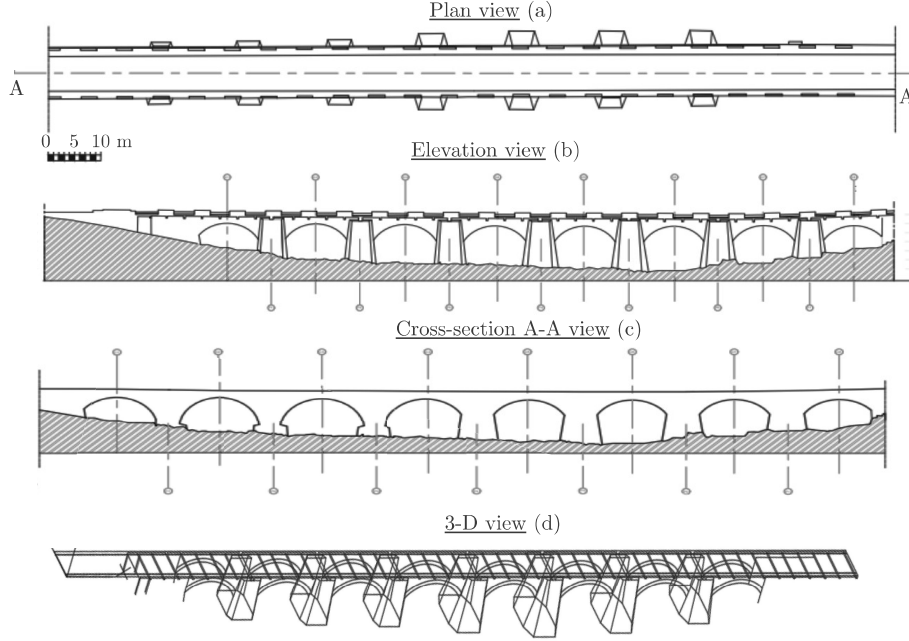


Figure 5: (a) Plan, (b) Elevation, (c) Cross-section and (d) 3-D views of the bridge.

At this point, further information regarding the geometry of the structure can be provided:

- Arches: They all have a similar average span of approximately 11.70 m. The smallest arch (on the far right) has a span of 11.17 m, while the largest (arch number 4) has a span of 12.48 m.
- Piers: There is a notable difference in height between the piers near the abutments, which have a height of approximately 8.50 m, and the central piers, which rise to around 10.80 m from the foundation level.
- Reinforced concrete beams and slab: The beams have a spacing of about 3.20 m with a cross-section of 30 cm by 30 cm, although not all beams follow this pattern. The beams supported by the piers have a double width equal to 60 cm. As for the slab, it has a thickness of about 30 cm and a width of approximately 90 cm.

4. Current state of the structure

In this section, the present state of the structure will be examined, including an assessment of the knowledge level and mechanical parameters of the essential components that comprise the masonry structure.

The structure's overall condition indicates advanced deterioration, characterized by a notable presence of cracks. The irregular geometry of the structure can be traced back to the 20th century when pier widening was introduced, resulting in a reduction of the arch spans. However, it should be noted that this modification was implemented selectively, affecting only half of the arches on the eastern side of the bridge.

130 The bridge does not appear to be in immediate danger regarding overall structural integrity. However, the interventions carried out in the 1980s, specifically the insertion of reinforced concrete elements, have deteriorated significantly. Many of these elements show the detachment of the cementitious conglomerate, exposing the reinforcement to the detrimental effects of weathering. Another essential aspect to consider is the presence of invasive vegetation and moisture stains caused by weather-related factors and leaching, as
 135 the water drainage system is inefficient. In some areas of the structure, a visual analysis of the elements is impossible as they are covered by a cementitious mortar, concealing the actual condition of the structure, see Figure 6.

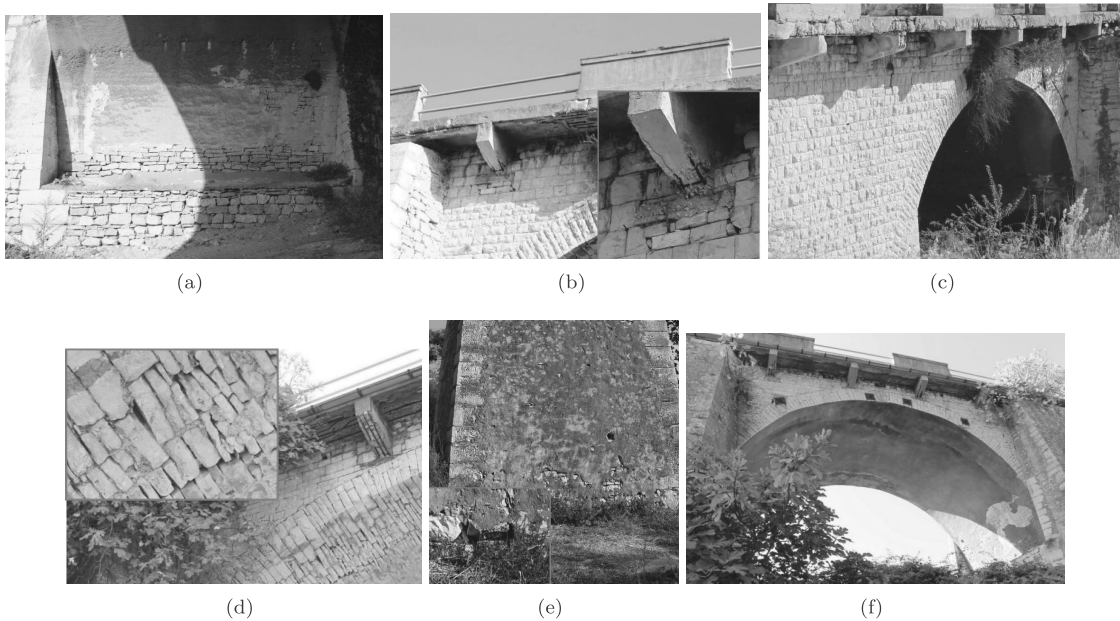


Figure 6: (a) Partially covered pier with cementitious mortar; (b) Current state of reinforced concrete elements; (c) Vegetation in the tympanums; (d) Current state of masonry elements; (e) Current state of a pier; (f) Transverse connection through chains.

The reinforced concrete structure's components display a substantial cracking pattern and widespread deterioration. There are clear signs of the cementitious mortar detaching, exposing corroded reinforcement.
 140 Cuts have been made in the tympanums to accommodate the placement of the reinforced concrete beams, resulting in stress concentrations and subsequent cracking.

The retaining devices, such as the concrete and local stone walls and the steel handrails, show no degradation. There is significant heterogeneity in the masonry structure. They are constructed using squared, roughly hewn stone blocks, with finished edges achieved through horizontal chiselling and generally staggered vertical joints. This typology is found in the tympanums, the arches' outercourse, and the abutments' cladding. Figure 6 highlights significant degradation and a pronounced cracking pattern. The interventions introduced in the late 1980s aimed to address this issue by implementing a transverse connection. However, it should be noted that this connection was not applied to all of the arches. Over time, the piers have changed geometry. The cross-sectional shape has been substantially modified, introducing a tapering effect from bottom to top, with a more extensive base starting from the foundation level. However, these interventions have resulted in significant structural irregularities, as all the piers exhibit varying degrees of slenderness.

The cementitious mortar does not cover most pier widening, exposing the material to atmospheric conditions. Additionally, the end stones are superior to those used in the central portion. In particular, focusing on one of these pier widening interventions, aside from the degraded state of the cementitious mortar, the undercutting at the base of the stone material composing this fundamental element is highlighted. Nevertheless, this does not affect the functionality of the structural component, as it is a minor defect compared to the overall structure scale.

Another type of intervention carried out on the masonry is the insertion of chains in the arches, as the longitudinal cracking pattern is quite pronounced. The transverse connection provided by the chains aims to counteract the destabilization effects associated with the existing cracking, which can generally be attributed to inadequate load distribution. Furthermore, moisture stains result from inadequate water drainage on the roadway platform. The presence of vegetation leads to moisture accumulation within the structure, exerting pressure on the tympanums of the structure.

In summary, the structural components of the examined structure show extensive deterioration.

- Arches: There is a moderate cracking pattern, with longitudinal cracks sometimes controlled by chains.
- Piers: There is no significant cracking pattern, although there have been cases of partial expulsion of stone material.
- Tympanums: There is a notable cracking pattern due to the absence of beams improving the load distribution.
- Reinforced concrete elements: They present the most critical condition within the structure despite

being the most recently introduced elements.

5. Structural survey and mechanical parameters identification

Several tests were conducted to investigate the internal morphology of the masonry, identifying homogeneous material areas and masonry texture, and locating connection elements such as dowels. Generally, #13 endoscopic tests, #3 extracted samples and #5 penetrometer tests at the level of the mortar between the bricks of the vault and of the pier were performed. Then, the specimens were tested in laboratory where compression tests were conducted for the evaluation of the masonry strength and density. #8 Endoscopic investigations were carried out in the arches (from ED6 to ED13). Among these tests, #5 exhibited a highly similar stratigraphic profile. The observed profile consisted of an initial layer measuring 20 cm in thickness, comprised of limestone rock, followed by filling material. Notably, test ED8 was performed at a height that facilitated the assessment of the actual vault thickness, which was determined to be 80 cm, see Figure 7.

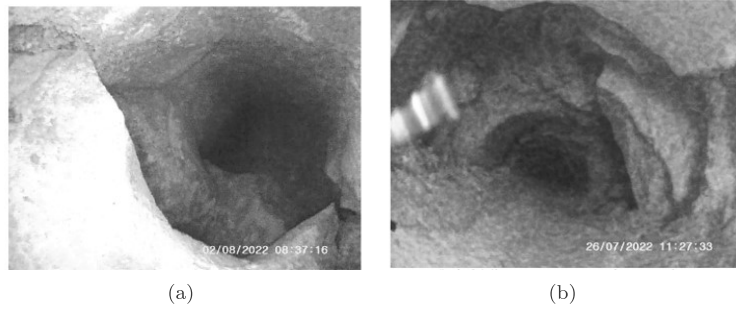


Figure 7: Endoscopic examination of the (a) arch and (b) pier.

Hence, a uniform thickness of 80 cm is adopted to define the structural model. The remaining #5 endoscopic tests conducted on the piers (ED1 to ED5) reveal significant heterogeneity in the stratigraphic profile. Even in the pier widening interventions with variable cross-sections, rubble-filled masonry is observed above 100 cm from the foundation level. As determined from the extracted samples, the mortar exhibits a compressive strength of approximately 0.25 MPa. Regarding the bridge deck, the extracted cores exhibit a layer of bituminous conglomerate with a thickness of approximately 15 cm. Beyond this layer, filling material is present, indicating the absence of a reinforced concrete slab, which is only present in the cantilevered sections.

To assess the design compressive strength, the corrective coefficients are applied to reduce the measured

compressive strength, as described in Equation 1:

$$f_d = \frac{f_m}{\gamma_M \text{FC}} \quad (1)$$

where γ_M represents the safety factor and is set to 3, and FC is the confidence factor based on the assumed structural knowledge and is taken as 1.2, following the recommendations of the Italian regulations [54].

Table 2: Mechanical properties of masonry components where f_m is the compressive strength, f_{vo} is the shear strength, E is the Young’s modulus and ν is the Poisson’s ratio.

Component	f_m [MPa]	f_{vo} [MPa]	E [MPa]	ν
Arc Masonry	3.2	0.065	1740	0.15
Pier Masonry	2	0.043	1230	0.15
Tympanum Masonry	7	0.105	2850	0.15

Table 3: Additional information on masonry components, where γ is the specific weight.

Component	γ [kN/m ³]	Correction Coefficient
Arc Masonry	21	1.8
Pier Masonry	20	1.44
Tympanum Masonry	22	-

Table 4: Design strength values for masonry components, where f_d is the design compressive strength.

Component	f_d [MPa]
Arc Masonry	1.58
Pier Masonry	0.8
Tympanum Masonry	1.94

Tables 2-4 present important information about different masonry components’ mechanical properties and design strength values.

Table 2 presents the mechanical properties of the masonry components, including the compressive strength (f_m), shear strength (f_{vo}), Young’s modulus (E), and Poisson’s ratio (ν). Table 3 provides additional information on the masonry components, specifically the specific weight (γ) and the correction coefficients (Table C8.5.II in NTC 2018 [54]). If technical construction details are implemented at the preliminary stage of the construction, correction coefficients can be adopted to improve the masonry’s strength and stiffness characteristics.

Table 4 focuses on the design strength values for the masonry components, specifically the compressive design strength (f_d). This table summarizes the design strength values for arc, pier, and tympanum masonry.

6. Load analysis and structural verifications

The actions to be considered include permanent structural, permanent non-structural, variable, and seismic actions. Table 5 lists the material densities and additional loads.

Table 5: Material densities and additional loads.

Material/Load	Density/Value
Reinforced Concrete	25 kN/m ³
Masonry (Arches)	21 kN/m ³
Masonry (Tympanums)	22 kN/m ³
Road Pavement	2.5 kN/m ³
Fill Material (Piles)	22 kN/m ³
Vehicle Restraint System	13.8 kN/m

According to Italian regulations, load combination is crucial for conducting ultimate limit state verifications [54]. For traffic loads, the Italian regulations recommend determining the notional lanes defined as the strip of the carriageway, which is deemed to carry a line of cars and/or lorries. In the considered bridge, the total deck width is 8.85 m, with 6.45 m dedicated to variable traffic actions, while hard shoulders of 1.20 m, placed at the right and left side of the carriageway, are intended for the passage of pedestrians. This suggests the presence of two notional lanes of 3.00 m and a remaining area of 0.45 m.

The following load combinations are considered and numbered according to Italian regulations.

- Load model 1 (LM1): The load scheme consists of concentrated loads placed on two tandem axles, applied over square tire prints (0.40 m x 0.40 m), and uniformly distributed loads. The axle load consists in 300 and 200 kN for the first and second lanes, respectively. The distributed load was equal to 9 kN/m² and applied only to the unfavourable parts of the influence surface, longitudinally and transversely. Breaking loads equal to 900 kN have been also modelled and involved in the final envelope for the detection of the critical loading scenario.
- Load model 5 (LM5): Represents the packed crowd load, including dynamic effects, with a nominal intensity of 5 kN/m².

The following verifications are carried out:

- Static Analysis:
 - Local static analysis: Assess stress states to predict the potential development of global hinged mechanisms.
 - Global static verification: Create a model of a single arch, incorporating all key structural components to evaluate behaviour and stress states.
- Dynamic Analysis:
 - Global seismic analysis: Capture the bridge's response under seismic loads.

- Local seismic analysis: Assess potential local kinematic mechanisms.

Regarding masonry structures, the safety assessment considers the ultimate limit state (ULS). The static analysis of the arches employs limit analysis theorems and assumes that masonry does not resist tension, no sliding occurs between the masonry blocks comprising the vault, and failure arises from the loss of equilibrium between parts, not material crushing.

Within the ULS, two conditions are considered:

- Structural resistance limit state (STR): Evaluate the maximum normal stresses to check the crushing resistance of the masonry arch blocks. For this structural verification, symmetric loading combinations have been assumed as the most critical configurations (see [55] and [56]).
- Equilibrium limit state (EQU): J. Heyman's approach [5] is employed for stability verification, assuming $\alpha > 3$, where α is defined in Equation 2. It must be verified that the pressure distribution curve is confined to the vault cross-section. In this case, asymmetric loading scenarios have been investigated since they were proved to be the most critical for the assessment of the collapse mechanism associated to the first kinematics for a proper plastic hinges configuration [57].

The equation representing the introduction of the parameter α for the EQU verification is as follows:

$$\alpha = \frac{t/2}{e_{\max}} \quad (2)$$

where t is the thickness of the vault, and $e_{\max}$ is the maximum eccentricity of the normal force.

7. Results of the static analysis

In the following sections, the definition of the FE model adopted for the local and global analyses will be described. In particular, for the static analysis, both analyses will be conducted with specific regard to the most critical span of the bridges. With this aim, span #4 has been selected due to the extensive damage observed during the survey at the level of either the vault and the pier, as well as for the high value of shallowness exhibited. The hypothesis of no interaction between consecutive modules is justified by piers with dimensions (i.e. 8.0x8.0 m) of the same order of the arch span (about 11 m). With the purpose of modeling the presence of the adjacent vaults to the investigated module, the longitudinal displacement has been constrained.

7.1. Local analysis

For the local static verification, the Aedes © Software [58], was adopted. A 3D model of the arch has been modelled by a shell size of 20 cm and a unit-width strip was considered to represent the tandem load action of a single lane, specifically the lane with the heaviest load. In this way, the critical section of the arch has been identified. This combination assumes two axis loads equal to 300 kN, of which 150 kN applied to the right-hand side, and 150 kN on the left-hand side, with respect to the centroid line of the notional lane. Therefore, the tandem is divided into two concentrated forces of 150 kN each, spaced longitudinally by 1.2 m and transversely by 2 m.

According to the Italian regulation, two load combinations will be used at the STR ultimate state: the symmetric combination at the keystone with permanent and variable loads. "Symmetric" refers to the tandem system positioned symmetrically to the arch's keystone (i.e., LM1). This load condition is especially critical for verifying structural strength. The other critical combination is the uniform crowd-loading scenario where 5 kN/m^2 acts over the entire module's decks according to the LM5.

In arch bridges, the haunches are the arch cross-sections between the crown and the springing line. Conversely, the springing section is cross-section where the arch curve begins to rise from the piers. It corresponds to the transition between the vertical support and the ascending curve of the arch.

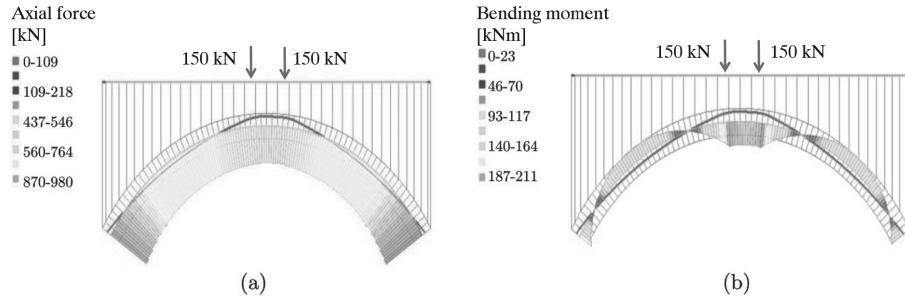


Figure 8: Diagram of the normal stress (a) and bending moment (b) for the symmetric-Tandem load combination at the STR limit state.

Figure 8 plots the diagram of the normal stress and bending moment for the symmetric load combination in the local verification where the symmetric-tandem load scenario has been investigated. The normal stress remains consistently compressive, reaching its maximum value at the springing section. Conversely, the bending moment peaks at the keystone and exhibits local maxima at the haunches and springing section.

Figure 9 plots the normal stress and bending moment diagram for the uniform crowd-loading combination in the STR local verification. The normal stress is always compressive, with the maximum value at the

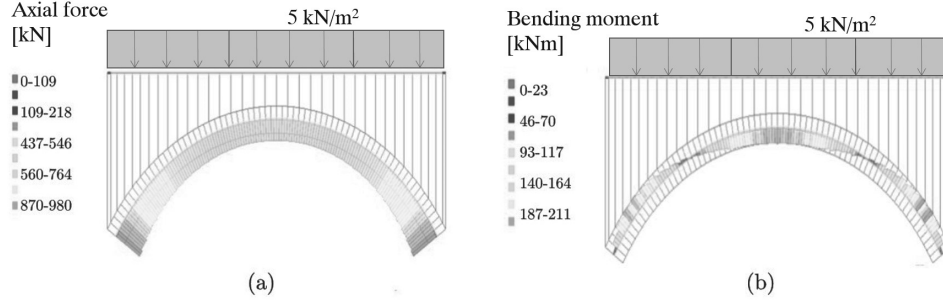


Figure 9: Diagram of the normal stress (a) and bending moment (b) for the symmetric-uniform crowd load combination at the STR limit state.

springing section, while the bending moment peaks at the keystone and has local maxima at the haunches.

With specific regard to the STR ultimate state verification, it is worth noting by a preliminary comparison
 280 between the results obtained by each loading scenario that the concentrated tandem forces at the top of the vault lead to significant normal stresses close to the springing sections of the arch, and a relevant positive bending moment at the mid-span.

Table 6: Maximum vs verification compression stresses at the keystone, haunches and springing sections for the symmetric-Tandem load and the uniform crowd-load combinations.

Position	Maximum stress [MPa]	Design stress [MPa]	Satisfied
	Symmetric-Tandem load combo		
Keystone	4.4	1.58	No
Haunches	2.93	1.58	No
Springing sect.	3.02	1.58	No
	Uniform crowd-load combo		
Keystone	0.77	1.58	Yes
Haunches	0.97	1.58	Yes
Springing sect.	0.91	1.58	Yes

Table 6 provides information on the maximum compression and verification stresses at the keystone, haunches, and springing sections for the investigated loading scenario (i.e. symmetric tandem and uniform
 285 crowd load). The maximum stress values are compared to the verification stress values to determine if the verification is satisfied.

For the symmetric tandem load, the maximum stress values at the keystone, haunches, and springing sections are all higher than the verification stress of 1.58 MPa. This indicates that the verification at the Structural resistance limit is unsatisfactory, as noted in the red highlighted cells.

290 On the other hand, for the uniform crowd load, the maximum stress values at the keystone, haunches, and springing sections are all lower than the verification stress. This means the verification at the STR is satisfied, as indicated by the green highlighted cells.

As mentioned above, with specific regard to the Equilibrium limit state, the most challenging condition for the asymmetric combination is the equilibrium verification. Coherently with the loading scenario provided for the STR ultimate state, two asymmetric combinations have been modelled: in the first one, the tandem system load has been placed asymmetrically with respect to the arch's keystone (permanent and variable loads have been also included), while in the second one, only half of the bridge span has been loaded with the same uniform crowd-load adopted in the STR verification

The pressure distribution curves for the asymmetric-tandem load and asymmetric-uniform crowd load combinations are shown in Figure 10.

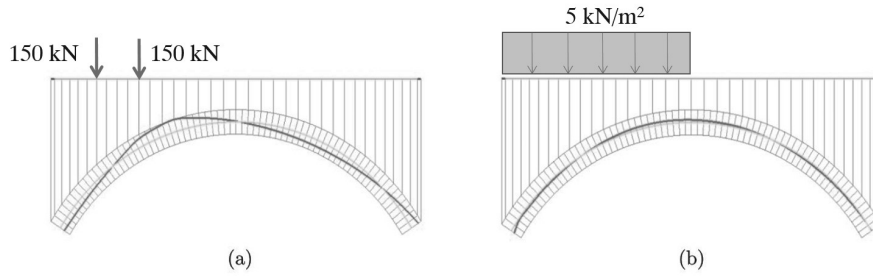


Figure 10: Pressure distribution curve for the asymmetric-tandem load (a) and asymmetric-uniform crowd load (b) combinations for EQU verification.

Referring to the asymmetric-tandem load scenario, the pressure distribution curve lies within the section but outside the central core of inertia. The geometric coefficient $\alpha = 1.32$ is not greater than the limit (i.e., 3), indicating unsatisfactory verification. In the case of the asymmetric-uniform crowd load scenario, the pressure curve lies within the section, confined within the central core of inertia, with a geometric coefficient value of 5.6. Consequently, the verification is satisfied. Additionally, by observing Fig.10-(b), it is evident that there is a slight difference between the pressure curve and the barycentric line of the section which highlights a near-pure axial stress condition acting into the arch.

7.2. Global analysis

The global structural model was implemented in Midas CIVIL © software [53] and shown in Figure 11. It encompasses the entire structure, including its components, such as arches, piers, abutments, and decks. It enables the analysis and evaluation of the structure's overall behaviour and response under different loading conditions. The global analysis considers the interaction and distribution of forces and moments throughout the structure. Various limit states are examined during the global analysis, including the equilibrium and resistance limit states. The equilibrium limit state ensures the stability of the structure as a rigid body by verifying the equilibrium of forces and moments. It assesses the structure's load-carrying capacity and

load distribution. On the other hand, the resistance limit state focuses on the maximum stress levels and the capacity of structural elements to withstand those stresses. It checks the strength and integrity of components to prevent failure caused by excessive loading.

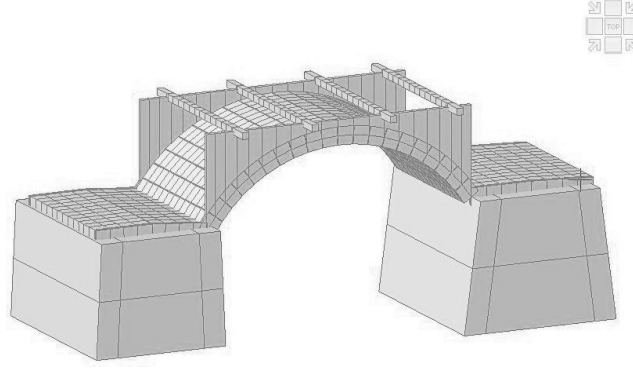


Figure 11: Global structural model.

The structural model includes the piers, vault, tympanums, and reinforced concrete (RC) beams. The
 320 piers are represented by a "beam" model with fixed supports at the base, as they are founded on piles. As
 observed by the geometric survey, a hole-cross-section has been assigned and the internal pressure of the soil
 against the pier's masonry has been evaluated. The vault is modelled as a "shell" element, extending beyond
 the springing level to ensure continuity between the piers. Additionally, to guarantee a correct transmission
 of the stress from the vault to the piers, rigid links have been adopted and all the nodes belonging to the
 325 top level of the piers have been fully-connected to the springing section of the arch. The same approach has
 been used for connecting the vault with the vertical shell element of the tympanum. The tympanums have
 a thickness of 20 cm and support the RC beams, represented as "beam" elements.

The presence of soil as filler material between the deck and the vault or at the inner part of the pier, has
 been modelled as equivalent area loads or as pressure loads depending on the case. Therefore, the weight of
 330 the soil has been converted in mass by using the *Mass Source* panel of the software such that it would be
 considered in the static and dynamic analysis.

It is essential to avoid disconnections to ensure the collaboration of all elements within the structural system.
 This can be evaluated through a global analysis that considers only the self-weight of the structural elements
 in the model. Figure 12 illustrates the model's response to self-weight loading which is coherent with the
 335 expected response and can be assumed as a preliminary global validation of the model.

Two load combinations are also considered in this analysis: symmetric and asymmetric. Figure 13
 illustrates the displacement and stress distribution of the symmetric load combination.

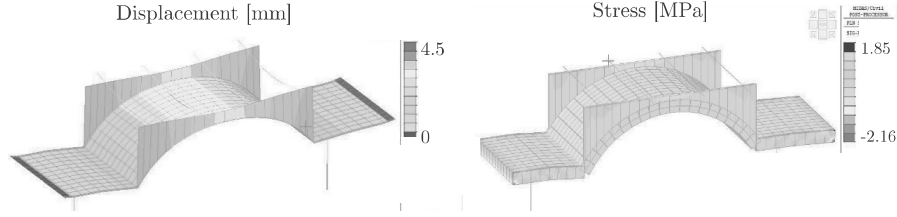


Figure 12: Vertical displacement (along Z-axis) and axial stress distribution (along the x-axis of the element local system) due to self-weight.

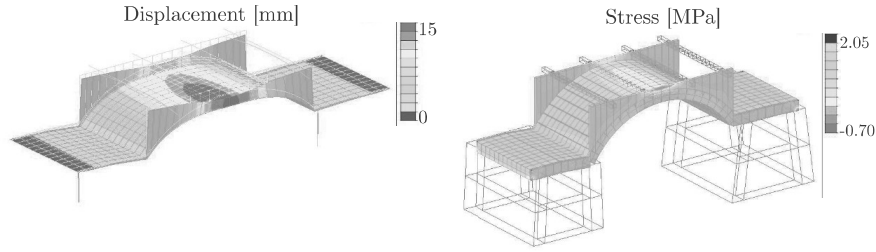


Figure 13: Vertical displacement (along global Z-axis) and axial stress distribution (along the x-axis of the element local system) due to symmetric load combination.

By conducting a detailed examination of the structure, the stress state of the vault can be analyzed. For stress verification, a linear variation of the stress state is assumed within a specific section, enabling the adoption of Navier's formula for sections experiencing flexural stress. Since masonry does not withstand tensile stress, a partial section is considered to evaluate an increased compressive stress state. The final stress verification does not meet the necessary criteria, as indicated below:

$$R_d = 1.58 \text{ MPa} > \sigma_{\max} = 2.72 \text{ MPa} \quad (3)$$

where R_d is the design stress and $\sigma_{\max}$ is the maximum stress from the model.

Figure 14 shows the displacement and stress distribution due to the asymmetric load combination.

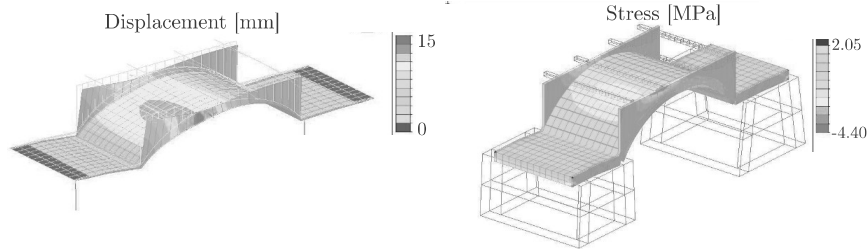


Figure 14: Displacement and stress distribution due to the asymmetric load combination.

In this verification, the stresses acting on the structure are significant, leading to the non-compliance

with the following inequality:

$$R_d = 1.58 \text{ MPa} > \sigma_{\max} = 1.72 \text{ MPa} \quad (4)$$

Figure 15 shows the stress distribution due to symmetric and asymmetric load combinations in a pier. As expected, the piers are entirely compressed. Moreover, in the symmetric loading combination, the presence of a small difference in the height of the piers leads to an asymmetric stress scenario. In both loading scenarios, the range of compression stresses variability in the piers is similar, as reported by the legend of the plot. Moreover, the effectiveness of the rigid connections adopted for the interaction modelling of the vault and the piers is demonstrated by the uniform stress distribution exhibited at the top of the pier.

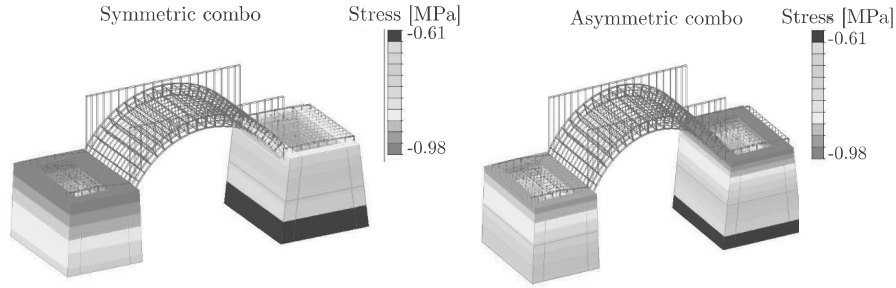


Figure 15: Axial stress distribution into the piers due to symmetric and asymmetric load combinations.

In both the symmetric and asymmetric combinations, the stresses of 0.91 MPa and 0.98 MPa, respectively, exceed the assigned strength of the element, which is 0.8 MPa. In conclusion, the conducted verifications do not meet the requirements. Alternative approaches will need to be considered to satisfy them. Thus, it can be concluded that the regulations for new bridge construction are not met.

8. Results of the seismic analysis

This section presents the results of the seismic analysis conducted on the structure. The seismic analysis is an essential step in assessing the structural integrity and safety of the bridge under earthquake loading conditions. The analysis considers the structure's dynamic behaviour and evaluates its response to seismic forces.

8.1. Global analysis

Figure 16 depicts the global model used for the seismic analysis. The same modelling approach described in the previous section for the static analysis has been adopted. Additionally, all the mass and loads have been considered in the seismic combination for the purpose of the analysis.

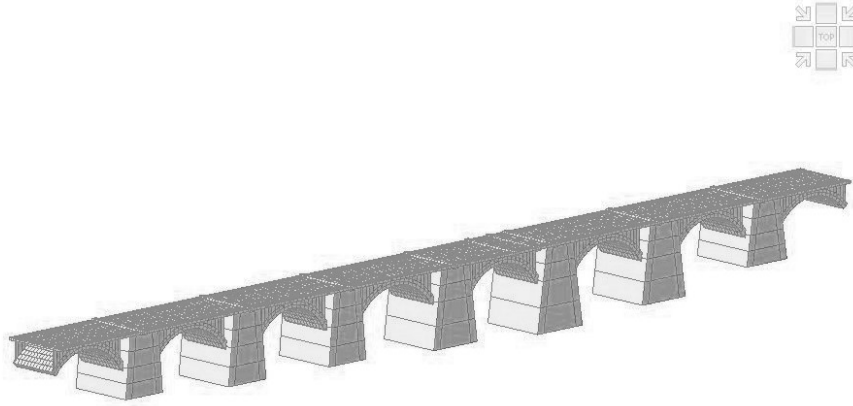


Figure 16: Global model for structural analyses.

The global finite element model comprehensively represents the entire structure, incorporating all its components and elements. The model captures the intricate interactions and behaviours of different structural elements, allowing engineers to assess the overall performance and safety of the bridge.

The seismic analysis was conducted using the method of linear dynamic analysis, employing the complete quadratic combination of effects. As per Italian regulations, the adopted structural factor was set to 1.5. Modes with a participation mass greater than 5% were considered, aiming to achieve 85% participation in each direction. This criterion poses a challenge for masonry structures since they were not originally designed to withstand seismic actions. In contrast, the situation is quite different for newly designed bridges as the mass is concentrated in the deck, making it easier to meet the regulatory limits.

The primary vibration modes for bridges predominantly exhibit flexural-torsional and pure flexural behaviour. Figure 17 illustrates the first four modes obtained from the modal analysis.

Table 7 lists the modal participation factor of the first 30 modes.

The expected seismic event corresponds to the design response spectrum specified by the Italian code for the specific site of interest, with a peak ground acceleration of 0.132g for the ultimate limit state and a return period of $T_r = 475$ years. In general, the stress state induced by seismic loads is much lower than that of static loads, indicating that the structure performs well under seismic action. Specifically, for arch number 7, which experiences the highest stress, the maximum stress value of 0.92 MPa does not exceed the design resistance stress of 2.39 MPa. Similarly, verifying the piers demonstrates that the maximum stress of 1.24 MPa is below the verification stress of 1.27 MPa, satisfying the safety requirements.

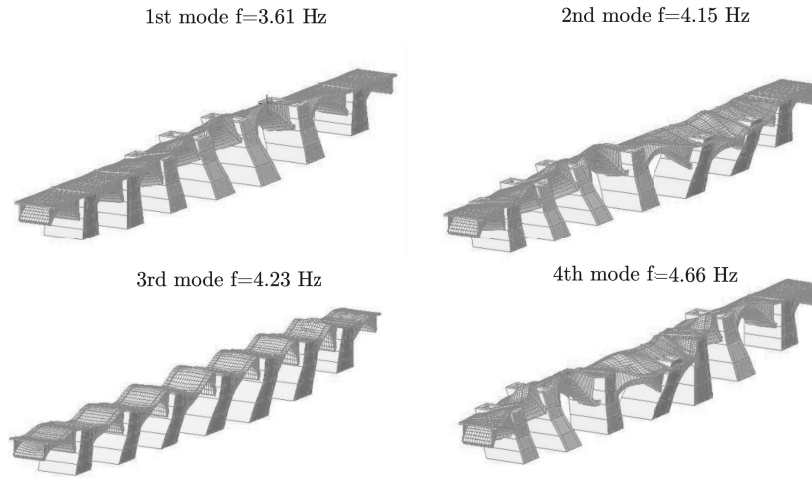


Figure 17: First four modes from the modal analysis.

Table 7: Table of participating modal masses, where TRAN and ROTN indicate translational and rotational degrees of freedom, X and Y are the longitudinal and transverse directions, while Z is the vertical direction.

Mode	T[s]	TRAN-X	TRAN-Y	TRAN-Z	ROTN-X	ROTN-Y	ROTN-Z
1	0.276	0	48.19	0	3	0	2.76
2	0.240	0	5.29	0	0.76	0	28.18
3	0.236	73.5	0	0	0	0.12	0
4	0.215	0	17.52	0	1.65	0	3.42
5	0.183	0	0	0	0.04	0	15.23
6	0.181	0.01	0	0.06	0	0	0
7	0.179	0.28	0	0.11	0	0	0
8	0.177	0.49	0	0	0	0.09	0
9	0.176	0	4.86	0	1.06	0	0.57
10	0.175	0	0.11	0	0	0	0
11	0.175	3.26	0	0	0	0.15	0
12	0.174	0	3.81	0	0.63	0	0.3
13	0.173	1.06	0	24.93	0	3.23	0
14	0.172	3.71	0	11.67	0	0.32	0
15	0.169	0.72	0	14.38	0	4.19	0
16	0.168	1.77	0	7.39	0	2.12	0
17	0.157	2.08	0	9.66	0	5.87	0
18	0.155	0	7.68	0	0.01	0	6.98
19	0.149	1.69	0	2.37	0	4.41	0
20	0.142	1.36	0	3.52	0	0.34	0
21	0.138	0	6.17	0	29.39	0	1.36
22	0.136	1.15	0	7.62	0	0.02	0
23	0.131	2.02	0	1.95	0	0.69	0
24	0.129	1.63	0	3.98	0	1.25	0
25	0.129	0	2.56	0	7.81	0	0.3
26	0.128	0.61	0	6.71	0	8.8	0
27	0.127	1.81	0	1.06	0	0.2	0
28	0.127	0	3.35	0	7.85	0	0
29	0.125	1.85	0	0.37	0	1.57	0
30	0.120	0.15	0	3.3	0	0.77	0

Masonry bridges exhibit high stiffness, which allows for minimal relative displacements under seismic acceleration unless a collapse mechanism is triggered. The formation of plastic hinges plays a critical role in generating such mechanisms. The seismic response of masonry bridges is often associated with an equilibrium problem, where geometric parameters significantly influence the behaviour. To investigate the behaviour of arches within a bridge, a kinematic analysis was performed to determine the minimum collapse multipliers required to activate the collapse mechanism. Therefore, in this paper, the plastic hinge concept refers to kinematic analyses, indicating the position where rotation is likely to occur under seismic forces. Therefore, the plastic hinges have been modeled as hinges, with no resisting bending moment.

The first step involves identifying the collapse mechanism by considering significant local mechanisms based on knowledge of seismic behaviour, crack patterns, or seismic-induced damage in similar structures. Next, the mechanism is modelled by introducing a sufficient number of hinges, representing areas where masonry exhibits cracking due to low tensile strength, enabling the rotation of connected rigid bodies. The load multiplier a_0 , responsible for the structural collapse, is then evaluated. According to the kinematic theorem (upper bound), this multiplier represents the minimum value that triggers collapse. Rigid blocks are subjected to various forces, including self-weight, vertical loads, proportional horizontal forces, external forces (e.g., transmitted through metal chains), and internal forces resulting from the interlocking effect of masonry blocks. Essentially, the evaluation of the a_0 has been conducted by considering structural permanent and no structural permanent loads (i.e. self-weight of the masonry and all the RC elements as well as the weight of the road pavement), while horizontal actions have been derived coherently with the suggestions provided by the NTC18 national standard in which the horizontal action is assumed equal to the 30% of the gravitational loads.

By assigning a generic rotation θ_k to a specific block k , displacements of applied forces can be determined based on the structure's geometry. The load multiplier a_0 is obtained using the principle of virtual work, which calculates the work generated by internal and external forces during a virtual motion. In the analyzed case, stout piers cause the structure to behave like a single-span arch, resulting in an asymmetric collapse mechanism that affects only the involved arch. This allows for the independent assessment of arches as separate structures, unlike bridges with slender piers, where a collapse mechanism would involve the entire system, leading to a global collapse mechanism.

Studies on collapse mechanisms of this type of structure emphasize increased vulnerability with higher deflection-to-span ratios and lower thickness-to-span ratios, with the former parameter having a more sub-

stantial influence due to its significant impact on the arch's geometry. Identifying the hinge positions that activate the mechanism among all possible combinations is crucial. The most probable position, yielding the minimum load multiplier, is selected. Hypothetical hinge locations are iteratively considered until the lowest collapse multiplier is determined; see Figure 18. The adopted iterative procedure mentioned above follows basically a trial-and-error approach in which the plastic hinges configuration is upgraded step-by-step until the detection of the minimum load multiplier. Then, the obtained solution is compared with the hinge configuration provided by performing an incremental static analysis through the FE model, where the horizontal loads (e.g. seismic action) are evaluated proportional to the gravitational ones.

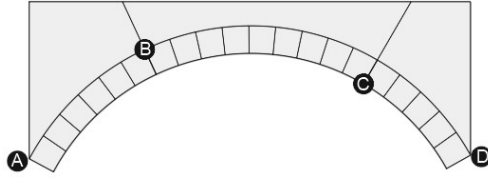


Figure 18: Labelling of the hinges in the masonry arch.

Table 8 shows the collapse multipliers and seismic analysis results. More in detail, the participating mass fraction, e^* , represents the fraction of the total mass, M^* , mobilized by the kinematic. a_0^* represents the acceleration at which the kinematic is effectively activated. It can be derived by:

$$a_0^* = \frac{a_0 g}{e^* F C} \quad (5)$$

where FC is a confident factor depending on the level of knowledge of the structure. Finally, a_g is the seismic design acceleration at the safety limit state where the behavior factor, q , is assumed equal to 2 according to the Italian regulation.

Table 8: Collapse multipliers and seismic analysis results

Parameter	Value
Collapse multiplier (a_0)	0.09
Participating mass fraction (e^*)	0.904
Activation acceleration (a_0^*)	0.0820 g
Seismic design acceleration (a_g)	0.132 g
Ratio of a_0^* to a_g	0.82

The verifications are not satisfied as the ratio between capacity and demand is 0.82.

9. Discussion and structural rehabilitation proposal

The structure is significantly deteriorated, posing challenges in meeting the verification requirements. Moreover, the knowledge level of the structure is rated as level 2, according to the Italian regulation, resulting in a 20% reduction in design strengths due to the confidence factor. Coherently with the Italian regulation, the reduction of loads acting on the structure is mandatory to ensure the bridge's usability for the following five years after the structural verification. Specifically, a loading condition equivalent to "vehicles" or "light vehicles", with a limit of 3.5 tons, has to be considered. It entails a uniformly distributed load of 2.5 kN/m^2 applied to areas that experience maximum stresses. Finally, to design the structural intervention and to recover the full functionality of the bridge, the standard loading conditions will be considered (without any type of traffic level limitation).

To enhance structural performance, it is necessary to consider the masonry characteristics, i.e. minimal tensile strength, and good compressive resistance. At the same time, the retrofitting actions must be calibrated to mitigate the structural deficiencies. Specifically, interventions improve the load-bearing capacity of masonry structures, e.g. the use of steel chains to prevent the tympanum overturning or to compensate the arch thrusts. Moreover, Concrete Reinforced Mortar (CRM) with composite fiber-reinforced materials or steel mesh represents effective retrofitting solutions to improve the shear and flexural behavior of the masonry. On the other hand, injections of fluidized hydraulic binders are adopted to fill voids, increase inter-particle friction, and improve compression resistance. Mass reduction is also beneficial, as masonry has a higher mass compared to reinforced concrete, leading to reduced forces during seismic events. Another intervention is the masonry confinement for bridge piers and columns, which enhances the compression resistance of the structural elements. This can be achieved using Fiber Reinforced Cementitious Matrix (FRCM) with high-strength steel mesh or other materials.

For all these reasons, three interventions are proposed:

1. Tie rods with a 2.15 m spacing and hollow circular cross-section with an external diameter of 42.4 mm and thickness of 4 mm;
2. Injections of fluidized hydraulic binder;
3. Application of CRM with a 50 mm thickness.

Introducing tie rods increases structural rigidity, specifically in the arch's ribs, which are susceptible to uplift. Two incremental factors can be considered for the design strength, 1.7 for the grout injection and 2 for the reinforced plaster.

Table 9: Calculation of design strength considering the structural interventions.

Element	Static Design Strength (f_d [MPa])	Seismic Design Strength (f_d [MPa])
Vault	2.67	4.00
Pier	1.60	2.40

Table 9 presents the design strengths calculated for the vault and piers, considering the beneficial effects of the interventions following the Italian standards. The design strengths are given separately for static and dynamic conditions.

The structural model has been revised to incorporate the interventions. Three tie rods have been placed into the arch ribs for each arch. These elements are modelled as "beams" made of S275 steel, with a circular cross-section and 2.15 m spacing. The reinforced plaster is modelled as a "shell" rigidly connected to the arch. It has a thickness of 5 cm, while the reinforced fibres have a thickness of 1 mm. The mechanical characteristics include a fresh mortar density of 2.05 g/cm³, a compression strength of 15 MPa (according to UNI 12390/3 [59], and tensile strength of 4.4 MPa. The static and seismic analysis confirms that the stresses on the structure are below the design values, satisfying all inequalities for the vaults and the piers, as shown in Tab.10.

Table 10: Structural verifications before and after the interventions.

Load	Combination	Structural Element	Demand stress [Mpa]	Design stress [Mpa]	Outcome Satisfyed
Before intervention					
Static	Symmetric	Vault	2.72	1.58	No
Static	Asymmetric	Vault	1.72	1.58	No
Static	Symmetric	Pier	0.91	0.8	No
Static	Asymmetric	Pier	0.98	0.8	No
Dynamic	Seismic	Vault	0.92	2.39	Yes
Dynamic	Seismic	Pier	1.24	1.27	Yes
After intervention					
Static	Symmetric	Vault	1.56	2.67	Yes
Static	Asymmetric	Vault	1.72	2.67	Yes
Static	Symmetric	Pier	0.85	1.6	Yes
Static	Asymmetric	Pier	0.93	1.6	Yes
Dynamic	Seismic	Vault	0.93	4	Yes
Dynamic	Seismic	Pier	1.22	2.4	Yes

Fig.19 displays the Von Mises stress in the most critical vault of the bridge before and after the interventions under the two load combinations, symmetric and asymmetric. Interestingly, as shown in Fig.19, the stresses do not decrease before and after the interventions. The reasons for successful structural verifications stay in the increase in capacity rather than the reduction of the demand.

Specific local verifications, such as delamination of the reinforced plaster and tensile failure of the tie rods, have also been successfully carried out, as shown in Tab.11. The delamination was analysed following

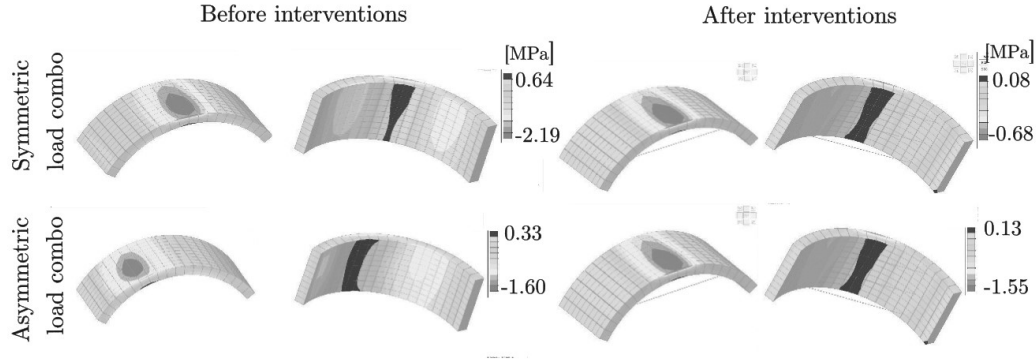


Figure 19: Von Mises stress in the most critical vault of the bridge before and after the interventions under the two load combinations, symmetric and asymmetric.

the recommendations in [60].

Table 11: Results of local verification in the tie rods and reinforced plaster.

Element	Failure mechanism	Capacity [MPa]	Demand [MPa]
Tie rod	Tensile failure	239	42
Reinforced plaster	Delamination	11.4	3.38

10. Cost and Environmental Lifecycle Analysis

It is essential to analyze the economic and environmental impacts concerning the improvements in structural performance. The interventions, such as using tie rods, reinforced plaster, and applying hydraulic binders, aim to enhance the structure’s load-bearing capacity, durability, and seismic resistance. Therefore, the cost and environmental impacts should be evaluated together with the benefits of the designed intervention in comparison with demolition and subsequent bridge reconstruction.

10.1. Cost estimations

The authors used the Italian price lists to estimate the costs of these interventions [61]. They considered material costs, labour expenses, and specialized expertise required for implementing the proposed measures. However, they did not include End-of-Life costs for material disposal and recycling due to a lack of reliable information. The cost estimate from the price list covers material procurement, construction processes, and other project management and quality control expenses. The estimated final cost amounts to €662,000 (€791.0/m²) (Figure 20). The higher costs are attributed to using hydraulic binders and reinforced plaster, which account for 52% and 39% total costs, respectively. On the other hand, lower costs are associated with installing steel ties and other equipment for seismic monitoring and on-site materials testing (Figure 21). The

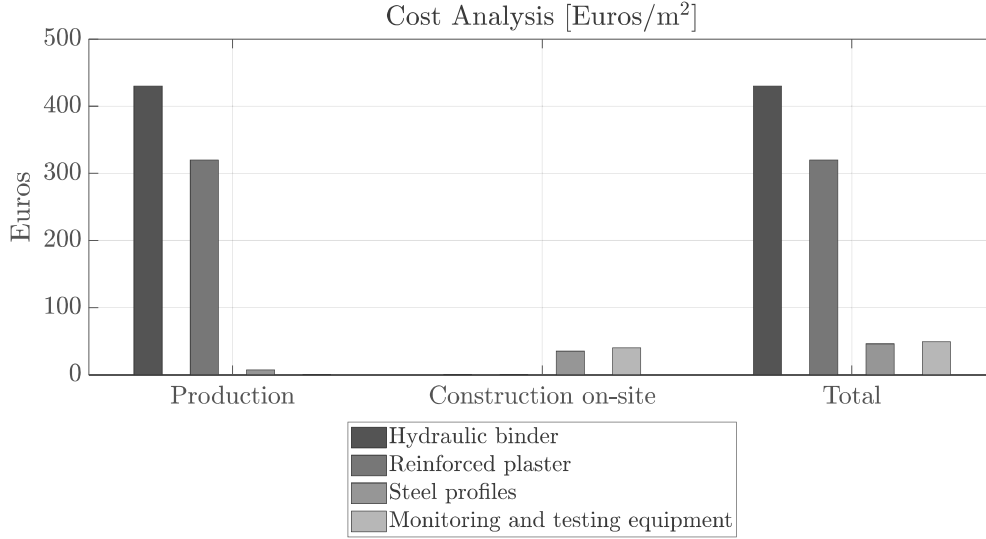


Figure 20: Cost analysis of the designed intervention. Results aggregated by lifecycle phases.

evaluated cost is significantly lower than the average required for the demolition and the reconstruction of a new bridge to be realized with the same static scheme and characteristics of the original one, by considering the current available technologies and technical practices.

10.2. Environmental analysis

For this study, an environmental analysis has been carried out. This step enables a better understanding of the abovementioned intervention's environmental potential, especially in comparison with demolition and new construction. The environmental analysis follows ISO 14040 [62] and ISO 14044 [63] standard (Lifecycle Assessment - LCA). For the calculation rules of environmental impacts, EN 15643 [64], and EN15798 [65] are considered. The environmental indicator is the Global Warming Potential total (GWP tot), as defined in EN15804+A2 [66]. Environmental information comes from the German public database for Construction works (Ökobau.dat - Version 2023-I [67]). Due to a lack of data, environmental datasets from Plastics Europe for Epoxy resin has been used [68]). The cradle-to-grave system boundaries entailed the raw materials extraction, the transport to the manufacturing site and manufacturing of binder and plaster premix as well as steel profiles and liquid epoxy resin (A1-A3 Modules [65]). For the hydraulic binder and reinforced plaster, the addition of further components to the premix and the water mixture to form a ready-to-use product are considered as activities occurring in the construction site (A5 Module [65]). For the same products, carbon storage due to the carbonation process is included in the calculation of B1 Module [65], as determined in

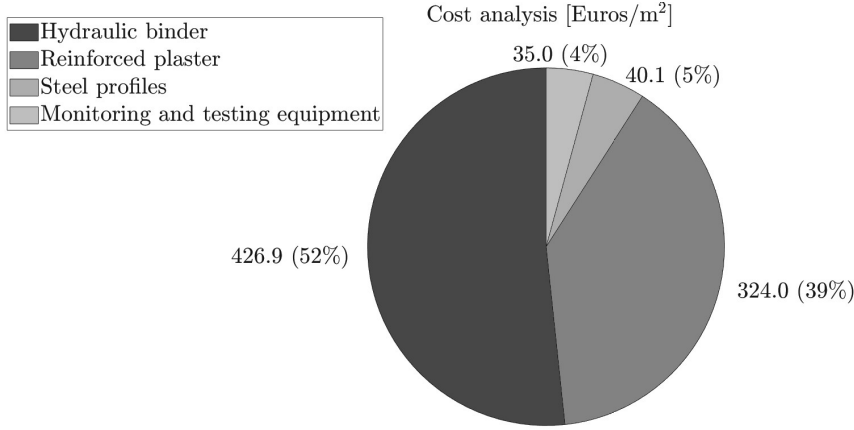


Figure 21: Cost analysis of the designed intervention. Results aggregated by material and equipment.

the used environmental datasets. The end-of-life (C3-C4-D Modules [65]) refers to the disposal process for mineral products (hydraulic binder, plaster) and steel products.

A GWP tot of 588.7 kg CO₂ eq./m² has been estimated for the global intervention. The highest impact value is related to CRM and its production (Figure 23). The used polymeric matrix is responsible for 11% of the total impact and the plaster for 86% (Figure 23). Despite the high impact values, it should be understood if the demolition of the existing building and a new building process could be more advantageous from the environmental point of view. Therefore, the derived value has been compared with a benchmark from literature [69], a new bridge construction compliant with current structural design codes with an average GWP_{tot} of 2400 kg CO₂ eq./m².

10.3. Remarks on Lifecycle analyses

The analysis carried out in this study and their results suggest that, in comparison with a new construction process, maintaining existing bridges and preferring retrofit or restoration measures can extend the bridge's service life and reduce greenhouse gas emissions. It should be underlined that bridge retrofit or restoration avoid construction waste, which in this study was not evaluated. From a social perspective, refurbishment could avoid longer infrastructure disruptions which could affect surrounding communities. In the presented environmental analysis, the main contributors of the total environmental impacts are highlighted to identify improvement potential. The obtained GWP value showed that CRM production has the highest contribution, due to energy and resource-intensive mixing plant activities (filling the storage or weighing bins, conveying the materials/mixing material into the mixer, mixing, conveying the finished product, packaging). Improving the efficiency of such processes in terms of energy and resources' consumption

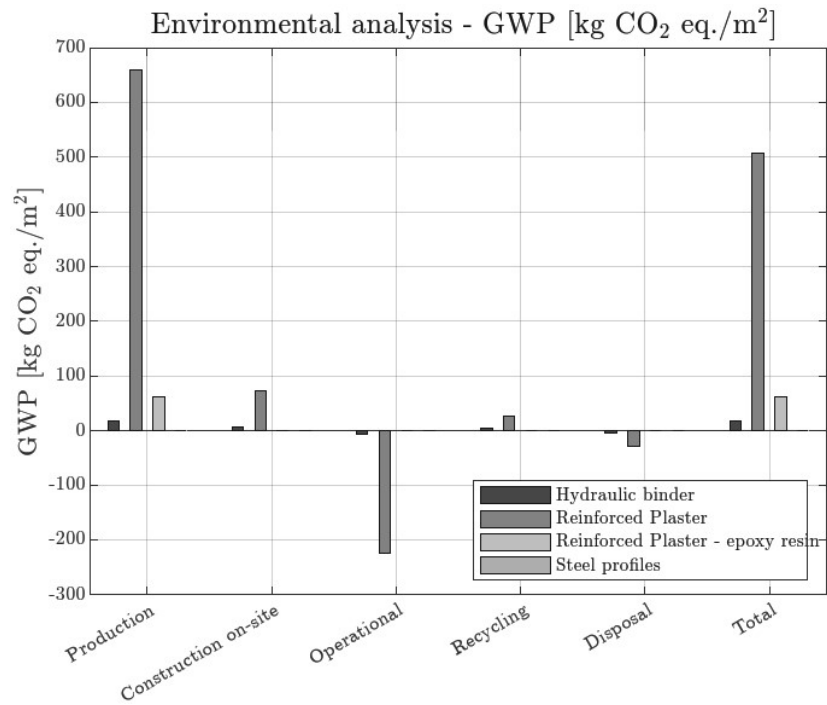


Figure 22: GWP of designed measure aggregated by lifecycle phases.

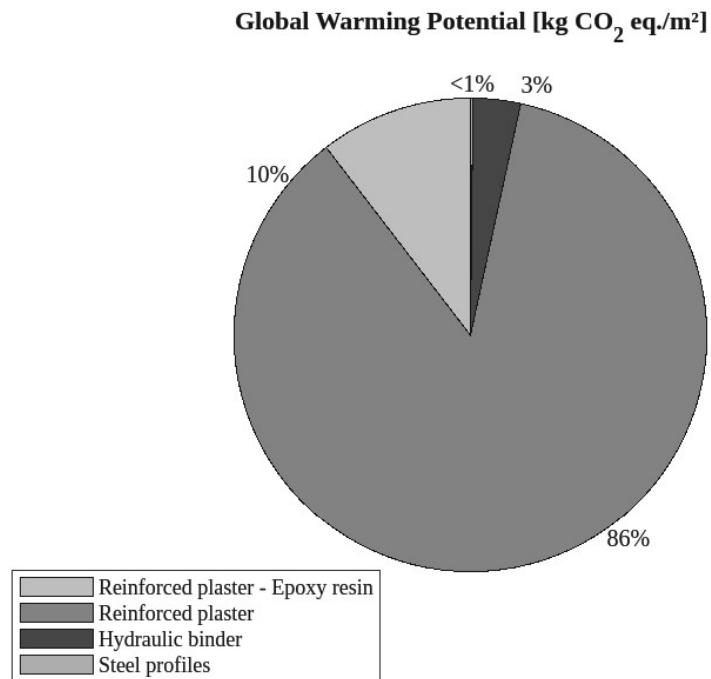


Figure 23: GWP of designed measure. Shares of used building materials.

as well as increasing the share of renewable energy sources are necessary strategies to reduce the environmental impacts of the CRM plaster. Epoxy resin production has also a significant contribution, which is derived from their large consumption of fossil fuels. In this regard, novel bio-based resins are currently being investigated as an environmental-friendly alternative [70] to be applied for CRM.

In this study, a traditional static approach has been applied to the used lifecycle analysis approach. Moreover, within the provided assessments, impacts related to construction operation and future required measures (B2-B5 Modules according to EN 15978 [65]) have been neglected. Both methodological choices can be justified from the context of the intervention design. The critical bridge structural performance under static loads leads to measures which should be immediately undertaken. For other contexts, whereas the remaining estimated service life is more prolonged and information regarding maintenance measures over construction service life is available, authors suggest other lifecycle analysis approaches, e.g. dynamic - probabilistic approaches as presented in Di Bari et al. 2019 [71]. Such approaches solve some shortcomings related to static approaches and support the decision-making process over the whole construction service life. This occurs by including more robust and reliable information about the required and foreseen construction maintenance and building parts replacement measures. Additionally, through the integration of the Probabilistic Seismic Hazard Approach (PSHA) and other events' occurrence probability functions directly in the constructions' lifecycle models, stakeholders are informed about environmental and economic effects related to unexpected and, nevertheless, non-negligible events. In conclusion, such approaches can provide valuable tools for a more-aware decision-making process in increasingly urgent resilient constructions.

11. Conclusions

This research focused on an in-depth examination of an existing masonry arch bridge, encompassing detailed surveying, modelling, structural verification, and proposed improvements. Accurate data on the bridge's condition was obtained using LiDAR and aerial survey techniques, while mechanical inspections identified critical areas requiring attention. The developed structural model underwent structural verifications, including seismic response analysis and identification of collapse mechanisms. The results confirmed that the bridge did not meet safety standards but also revealed specific vulnerabilities that needed to be addressed.

Based on the verification findings, structural improvements were proposed to enhance the bridge's performance and safety. These interventions involved introducing additional support elements, reinforcing critical areas, and utilising advanced materials to improve tensile strength and durability.

The findings of this study contribute significantly to the comprehension of modeling and structural verification for masonry bridges. By integrating detailed surveys, advanced modeling techniques, and thorough verification procedures, a comprehensive assessment of the bridge's condition is achieved, facilitating the identification of effective mechanical enhancement strategies.

Moreover, carrying out the economic and environmental impacts assessment of each material and component of the designed structural improvement measures, allowed the identification of main contributors, in which lies the highest improvement potential. With specific regard to the cost analysis, hydraulic binder and reinforced plaster represent the highest voices of the total cost, while the CRM components, due to the utilization of composite fibres, cover almost the entire assessed CO₂ eq.

Finally, this research introduces novelties by emphasizing two key aspects. Firstly, it conducts a detailed examination of a historic masonry bridge, exploring potential interventions to mitigate its vulnerability. This evaluation encompasses not only technological facets but also considers economic and environmental costs. Secondly, the research presents a comprehensive approach for ancient masonry bridges, assessing various intervention types, including the possibility of demolition and reconstruction. The proposed methodology for a new design approach prioritizes solutions that are not only mechanically sound but also strike a balance with cost considerations, such as economic and sustainability factors. In this context, the contribution offers a perspective where the optimal solution is not exclusively based on mechanical performance but also considers broader lifecycle criteria.

Future research efforts could focus on monitoring the implemented interventions. Accordingly, cost and environmental impact assessments need to include all required measures during the whole construction lifecycle model, with particular attention on the future potential impacts related to maintenance, components replacements and refurbishments. This will allow a holistic evaluation of the long-term performance of a measure and will further support stakeholders during the decision-making process.

Declarations of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgements

The authors would like to express their sincere gratitude to Prof. Rita Greco (Politecnico di Bari) for providing the technical data for the research and Eng. Francesco De Luca for the support during the modelling

590 phase. The research leading to these results has received funding from (i) the European Research Council
under the Grant agreement ID: 101007595 of the project ADDOPTML, MSCA RISE 2020 Marie Skłodowska
Curie Research and Innovation Staff Exchange (RISE); (ii) the European Union HORIZON-MSCA-2021-
SE-01, grant agreement No: 101086413, ReCharged - Climate-aware Resilience for Sustainable Critical
and interdependent Infrastructure Systems enhanced by emerging Digital Technologies; (iii) the Deutsche
595 Forschungsgemeinschaft (DFG, German Research Foundation) under Germany's Excellence Strategy—EXC
2120/1—390831618.

Availability of data and material

All data, models, or codes supporting this study's findings are available from the authors upon request.

References

- [1] K.-E. Kurrer, The History of the Theory of Structures: From Arch Analysis to Computational Mechanics, 2009. doi: 10.1002/9783433600160.
- [2] A. Brencich, R. Morbiducci, Masonry arches: Historical rules and modern mechanics, *International Journal of Architectural Heritage* 1 (2) (2007) 165–189. doi:10.1080/15583050701312926.
- [3] G. Curioni, L’Arte di Fabbricare. Costruzioni Civili, Stradali, Idrauliche (1870).
- [4] J. Page, Masonry arch bridges, *Masonry Arch Bridges* (1993).
- [5] J. Heyman, The Masonry Arch (1982).
- [6] A. Castigliano, Theorie de l’équilibre des systemes elastiques et ses applications, *Théorie de l’Équilibre des Systèmes Élastiques et Ses Applications* (1879).
- [7] Z. Orbán, M. Gutermann, Assessment of masonry arch railway bridges using non-destructive in-situ testing methods, *Engineering Structures* 31 (10) (2009) 2287–2298, cited By 112. doi:10.1016/j.engstruct.2009.04.008.
- [8] G. Poleni, Memorie storiche della gran cupola del tempio vaticano, *Memorie Storiche Della Gran Cupola Del Tempio Vaticano* (1748).
- [9] E. Mery, Sur l’équilibre des voûtes en berceau, *Annales des Ponts et Chaussées* 19 (2) (1840) 50–70.
- [10] I. Lubowiecka, J. Armesto, P. Arias, H. Lorenzo, Historic bridge modelling using laser scanning, ground penetrating radar and finite element methods in the context of structural dynamics, *Engineering Structures* 31 (11) (2009) 2667–2676, cited By 135. doi:10.1016/j.engstruct.2009.06.018.
- [11] P. Fanning, T. Boothby, Three-dimensional modelling and full-scale testing of stone arch bridges, *Computers and Structures* 79 (29-30) (2001) 2645–2662. doi:10.1016/S0045-7949(01)00109-2.
- [12] M. Domaneschi, D. Sigurdardottir, B. Glišić, Damage detection on output-only monitoring of dynamic curvature in composite decks, *Structural Monitoring and Maintenance* 4 (1) (2017) 1–15. doi:10.12989/smm.2017.4.1.001.
- [13] M. Domaneschi, M. Limongelli, L. Martinelli, Multi-site damage localization in a suspension bridge via aftershock monitoring, *Ingegneria Sismica* 30 (3) (2013) 56–72.
- [14] M. Morgese, M. Domaneschi, F. Ansari, G. Cimellaro, D. Inaudi, Improving distributed fiber-optic sensor measures by digital image correlation: Two-stage structural health monitoring, *ACI Structural Journal* 118 (6) (2021) 91–102. doi:10.14359/51732994.
- [15] A. Brencich, D. Sabia, Experimental identification of a multi-span masonry bridge: The tanaro bridge, *Construction and Building Materials* 22 (10) (2008) 2087–2099. doi:10.1016/j.conbuildmat.2007.07.031.
- [16] C. Costa, D. Ribeiro, P. Jorge, R. Silva, A. Arêde, R. Calçada, Calibration of the numerical model of a stone masonry railway bridge based on experimentally identified modal parameters, *Engineering Structures* 123 (2016) 354–371. doi: 10.1016/j.engstruct.2016.05.044.
- [17] A. Bautista-De Castro, L. Sanchez-Aparicio, P. Carrasco-Garcia, L. Ramos, D. Gonzalez-Aguilera, A multidisciplinary approach to calibrating advanced numerical simulations of masonry arch bridges, *Mechanical Systems and Signal Processing* 129 (2019) 337–365. doi:10.1016/j.ymssp.2019.04.043.
- [18] P. Borlenghi, A. Saisi, C. Gentile, Determining and tuning models of a masonry bridge for structural assessment, *Lecture Notes in Civil Engineering* 200 LNCE (2022) 409–417. doi:10.1007/978-3-030-91877-4_47.
- [19] M. Solla, J. Caamaño, B. Riveiro, P. Arias, A novel methodology for the structural assessment of stone arches based on geometric data by integration of photogrammetry and ground-penetrating radar, *Engineering Structures* 35 (2012) 296–306, cited By 40. doi:10.1016/j.engstruct.2011.11.004.
- [20] N. Domede, A. Sellier, T. Stablon, Structural analysis of a multi-span railway masonry bridge combining in situ observations, laboratory tests and damage modelling, *Engineering Structures* 56 (2013) 837–849. doi:10.1016/j.engstruct.2013.05.052.
- [21] S. Cocking, S. Acikgoz, M. Dejong, Interpretation of the dynamic response of a masonry arch rail viaduct using finite-element modeling, *Journal of Architectural Engineering* 26 (1) (2020). doi:10.1061/(ASCE)AE.1943-5568.0000369.
- [22] B. Conde, L. Ramos, D. Oliveira, B. Riveiro, M. Solla, Structural assessment of masonry arch bridges by combination of non-destructive testing techniques and three-dimensional numerical modelling: Application to vilanova bridge, *Engineering Structures* 148 (2017) 621–638, cited By 102. doi:10.1016/j.engstruct.2017.07.011.
- [23] A. P. Piccolo, G. Longobardi, A. Formisano, Seismic vulnerability and consolidation by frp/frcm systems of a masonry school building in the district of naples, *Buildings* 12 (11) (2022). doi:10.3390/buildings12112040. URL <https://www.mdpi.com/2075-5309/12/11/2040>
- [24] N. Chieffo, A. Formisano, T. Miguel Ferreira, Damage scenario-based approach and retrofitting strategies for seismic risk mitigation: an application to the historical centre of sant’antimo (italy), *European Journal of Environmental and Civil Engineering* 25 (11) (2021) 1929–1948.
- [25] B. Orfeo, J. León, I. Lorenzo, L. Todisco, Reconstruction of a 19th century masonry bridge taking advantage of literature-based ancient techniques and current technologies, *International Journal of Architectural Heritage* (2022) 1–17.
- [26] F. Haile, J. Adkins, M. Corradi, A review of the use of titanium for reinforcement of masonry structures, *Materials* 15 (13) (2022) 4561.
- [27] L. Pelà, A. Aprile, A. Benedetti, Comparison of seismic assessment procedures for masonry arch bridges, *Construction and Building Materials* 38 (2013) 381–394. doi:10.1016/j.conbuildmat.2012.08.046.
- [28] V. Sarhosis, S. De Santis, G. de Felice, A review of experimental investigations and assessment methods for masonry arch bridges, *Structure and Infrastructure Engineering* 12 (11) (2016) 1439–1464, cited By 109. doi:10.1080/15732479.2015.1136655.
- [29] P. Zampieri, M. Zanini, F. Faleschini, Derivation of analytical seismic fragility functions for common masonry bridge types:

- methodology and application to real cases, *Engineering Failure Analysis* 68 (2016) 275–291. doi:10.1016/j.engfailanal.2016.05.031.
- [30] O. Saygılı, J. Lemos, Seismic vulnerability assessment of masonry arch bridges, *Structures* 33 (2021) 3311–3323. doi:10.1016/j.istruc.2021.06.057.
- [31] S. Gönen, S. Soyöz, Seismic analysis of a masonry arch bridge using multiple methodologies, *Engineering Structures* 226, cited By 39 (2021). doi:10.1016/j.engstruct.2020.111354.
- [32] P. Zampieri, C. Tetougueni, C. Pellegrino, Nonlinear seismic analysis of masonry bridges under multiple geometric and material considerations: Application to an existing seven-span arch bridge, *Structures* 34 (2021) 78–94. doi:10.1016/j.istruc.2021.07.009.
- [33] A. Formisano, M. Felitti, F. Oliveto, L. Mendicino, The robustness of reinforced concrete tied arch bridges: A case study, *Structural Concrete* 24 (4) (2023) 4504–4514. doi:https://doi.org/10.1002/suco.202200374.
- [34] L. Marcheggiani, F. Clementi, A. Formisano, Static and dynamic testing of highway bridges: A best practice example, *Journal of civil structural health monitoring* 10 (1) (2020) 43–56.
- [35] R. Silva, C. Costa, A. Arêde, Numerical methodologies for the analysis of stone arch bridges with damage under railway loading, in: *Structures*, Vol. 39, Elsevier, 2022, pp. 573–592.
- [36] F. Bencardino, R. Curto, V. Scavelli, Inspection and structural rehabilitation of an existing masonry arch railway bridge, *Applied Sciences* 13 (5) (2023) 2973.
- [37] J. León, B. Orfeo, L. Todisco, P. Miner, Design of stone masonry bridges in european treatises: part 2—the construction process, in: *Proceedings of the Institution of Civil Engineers-Bridge Engineering*, Thomas Telford Ltd, 2023, pp. 1–13.
- [38] G. Milani, P. Lourenço, 3d non-linear behavior of masonry arch bridges, *Computers and Structures* 110–111 (2012) 133–150, cited By 170. doi:10.1016/j.compstruc.2012.07.008.
- [39] A. Thavalingam, N. Bicanic, J. Robinson, D. Ponniah, Computational framework for discontinuous modelling of masonry arch bridges, *Computers and Structures* 79 (19) (2001) 1821–1830. doi:10.1016/S0045-7949(01)00102-X.
- [40] S. Tapkın, E. Tercan, S. Motsa, G. Drosopoulos, M. Stavroulaki, E. Maravelakis, G. Stavroulakis, Structural investigation of masonry arch bridges using various nonlinear finite-element models, *Journal of Bridge Engineering* 27 (7) (2022). doi:10.1061/(ASCE)BE.1943-5592.0001870.
- [41] F. Scozzese, L. Ragni, E. Tubaldi, F. Gara, Modal properties variation and collapse assessment of masonry arch bridges under scour action, *Engineering Structures* 199 (2019). doi:10.1016/j.engstruct.2019.109665.
- [42] J. George, A. Menon, Analytical fragility curves for displacement-based scour assessment of masonry arch bridges, in: *Structures*, Vol. 46, Elsevier, 2022, pp. 172–185.
- [43] O. Bouzas, B. Conde, J. C. Matos, M. Solla, M. Cabaleiro, Reliability-based structural assessment of historical masonry arch bridges: The case study of cernadela bridge, *Case Studies in Construction Materials* 18 (2023) e02003.
- [44] B. Pulatsu, E. Erdogmus, P. Lourenço, Comparison of in-plane and out-of-plane failure modes of masonry arch bridges using discontinuum analysis, *Engineering Structures* 178 (2019) 24–36, cited By 49. doi:10.1016/j.engstruct.2018.10.016.
- [45] F. Cannizzaro, B. Pantò, S. Caddemi, I. Calì, A discrete macro-element method (dmem) for the nonlinear structural assessment of masonry arches, *Engineering Structures* 168 (2018) 243–256, cited By 58. doi:10.1016/j.engstruct.2018.04.006.
- [46] A. Rahimi, S. B. B. Aval, M. Noori, V. Sarhosis, Z. Wu, A. Nikkhoo, W. A. Altabey, A simplified beam model for the numerical analysis of masonry arch bridges—a case study of the veresk railway bridge, in: *Structures*, Vol. 45, Elsevier, 2022, pp. 1253–1266.
- [47] F. Accornero, G. Lacidogna, A. Carpinteri, Evolutionary fracture analysis of masonry arches: Effects of shallowness ratio and size scale, *Comptes Rendus - Mécanique* 344 (9) (2016) 623–630, cited By 14. doi:10.1016/j.crme.2016.05.002.
- [48] F. Accornero, G. Lacidogna, A. Carpinteri, Medieval arch bridges in the lanzo valleys, Italy: Case studies on incremental structural analysis and fracturing benefit, *Journal of Bridge Engineering* 23 (7), cited By 12 (2018). doi:10.1061/(ASCE)BE.1943-5592.0001252.
- [49] J. Matos, V. Moreira, I. Valente, P. Cruz, L. Neves, N. Galvão, Probabilistic-based assessment of existing steel-concrete composite bridges – application to sousa river bridge, *Engineering Structures* 181 (2019) 95–110, cited By 15. doi:10.1016/j.engstruct.2018.12.006.
- [50] D. Kang, H. Lee, H. Park, I. Lee, Computing method for estimating strain and stress of steel beams using terrestrial laser scanning and fem, *Key Eng. Mater.* (2007) 347Cited By 1.
- [51] D. Oliveira, P. Lourenço, C. Lemos, Geometric issues and ultimate load capacity of masonry arch bridges from the northwest iberian peninsula, *Engineering Structures* 32 (12) (2010) 3955–3965, cited By 99. doi:10.1016/j.engstruct.2010.09.006.
- [52] Autodesk, Autocad software 2024.
URL <https://www.autodesk.it/products/autocad>
- [53] C. software house, Midas civil software.
URL <https://globalsupport.midasuser.com/helpdesk/KB/View/32609792-midas-civil-manuals-and-tutorials>
- [54] I. M. o. I. and NTC, Norme Tecniche per le Costruzioni. DM 17/1/2018, Gazzetta Ufficiale della Repubblica Italiana (2018) 1–198.
- [55] J. Heyman, *Structural analysis: a historical approach*, Cambridge University Press, 1998.
- [56] S. Huerta Fernández, *Mechanics of masonry vaults: The equilibrium approach* (2001).
- [57] J. Heyman, The safety of masonry arches, *International Journal of Mechanical Sciences* 11 (4) (1969) 363–385.
- [58] A. software house, Aedes, software per ingegneria civile.
URL <https://www.aedes.it/it/web/guest/azienda>
- [59] E. UNI, 12390-3. testing hardened concrete, *Compressive strength of test specimens* (2009).

- [60] M. d. I. e dei Trasporti, Consiglio superiore dei lavori pubblici linee guida per la classificazione e gestione del rischio, la valutazione della sicurezza ed il monitoraggio dei ponti esistenti, Ministero delle Infrastrutture e dei Trasporti, Rome (2020).
- [61] A. SOFTWARE, Primus.
URL <https://www.acca.it/software-computo-metrico-gratis?gclid>
- [62] International Organization for Standardization, Iso en 14040:2006 - environmental management- life cycle assessment - principles and framework (2006).
URL <https://www.iso.org/>
- [63] International Organization for Standardization, Iso en 14044:2006 - environmental management- life cycle assessment - requirements and guidelines (2006).
URL <https://www.iso.org/>
- [64] European Committee for Standardization, En 15643:2021 - sustainability of construction works. sustainability assessment of buildings and civil engineering works (2021).
URL <https://www.cencenelec.eu/>
- [65] European Committee for Standardization, En 15804:2020 - sustainability of construction works - environmental product declarations - core rules for the product category of construction products (2020).
URL <https://www.cencenelec.eu>
- [66] European Committee for Standardization, En 15978:2011 - sustainability of construction works. assessment of environmental performance of buildings. calculation method (2011).
URL <https://www.cencenelec.eu>
- [67] Bundesinstitut für Bau-, Stadt- und Raumforschung (BBSR), Ökobaumat, version 2023-i (2023).
URL <https://oekobaumat.de/>
- [68] PlasticsEurope, Eco-profiles plasticseurope's eco-profiles comprise life cycle inventory datasets (lci) and environmental product declarations (epd) for plastics. (2023).
URL <https://legacy.plasticseurope.org/en/resources/eco-profiles>
- [69] T. D. David Collings Dr, The carbon footprint of bridges, Structural Engineering International 32 (4) (2022) 501–506. doi:10.1080/10168664.2021.1917326.
- [70] J. Liu, L. Zhang, W. Shun, J. Dai, Y. Peng, X. Liu, Recent development on bio-based thermosetting resins, Journal of Polymer Science 59 (14) (2021) 1474–1490. arXiv:<https://onlinelibrary.wiley.com/doi/pdf/10.1002/pol.20210328>, doi:<https://doi.org/10.1002/pol.20210328>.
URL <https://onlinelibrary.wiley.com/doi/abs/10.1002/pol.20210328>
- [71] R. Di Bari, A. Belleri, A. Marini, R. Horn, J. Gantner, Probabilistic life-cycle assessment of service life extension on renovated buildings under seismic hazard, Buildings 10 (3) (2020). doi:10.3390/buildings10030048.